ACI STRUCTURAL JOURNAL TECHNICAL PAPER

Title no. 108-S45

Effect of Stirrup Anchorage on Shear Strength of Reinforced Concrete Beams

by Jason C. Varney, Michael D. Brown, Oguzhan Bayrak, and Randall W. Poston

The performance of improperly constructed beams is of particular concern in the repair of concrete structures. Not only is there uncertainty about the actual as-built strength and what measures may improve capacity, but also the presence of improper structural details may lead to litigation concerning what structural repairs are really necessary to restore perceived loss of strength. For the specific case of shear, there is value in knowing a reasonable and safe approximation of the shear capacity of improperly detailed beams.

To study the effect of improperly anchored stirrups on the shear strength of reinforced concrete beams, four 13 x 24 in. (330 x 610 mm) reinforced concrete sections were fabricated with varying proper and improper shear reinforcement details and loaded to failure. Current ACI 318 and AASHTO LRFD code provisions were used to compare the resulting failure loads from tests with calculated nominal capacities. The experimental results suggest that reinforcement anchorage, as tested, has no significant effect on the shear capacity of a reinforced concrete section.

Keywords: reinforced concrete; repair; shear; shear reinforcement; stirrup anchorage; strengthening.

INTRODUCTION

The performance of improperly constructed beams is of particular concern for the repair of concrete structures. Not only is there uncertainty about the actual as-built strength and what measures would improve capacity, but also the presence of improper structural details may lead to litigation concerning what structural repairs are really necessary to restore perceived loss of strength. Thus, there is value in knowing a reasonable and safe approximation of the shear capacity of improperly detailed beams. Proper understanding of the importance of shear reinforcement anchorage is necessary to design appropriate structural repairs. Whitlock Dalrymple Poston & Associates, PC, (WDP) has encountered questionable as-built stirrup details in the evaluation of existing buildings in the U.S. on two occasions: a 26-story residential high-rise in a highly active seismic zone on the Pacific coast (Building 1) and a public building in the Midwest (Building 2). Significant time and financial resources were consumed by various parties involved in the design, construction, evaluation, repair, and subsequent litigation of these projects to address the perceived deficiencies. An investigation of the ductile moment frame beams in Building 1 led to the discovery of closed stirrups lacking longitudinal corner bars (Fig. 1 and 2). Corner bars are required by ACI 318-08, Section 7.11.3,[1] to properly anchor shear reinforcement, and their absence raised questions regarding the as-built shear capacity of the beams.

Investigation of Building 2 uncovered a lack of code-required bends of center-leg stirrups around longitudinal bars on the tension side of the beam. It appeared as if the inclusion of these stirrups was nearly forgotten during construction and they were placed at the last minute (Fig. 3 and 4). A similar requirement in ACI 318-08, Section 12.13.2.1-2,[1] is present for single-legged stirrups, stating that each end must be properly anchored by being hooked around a longitudinal bar.

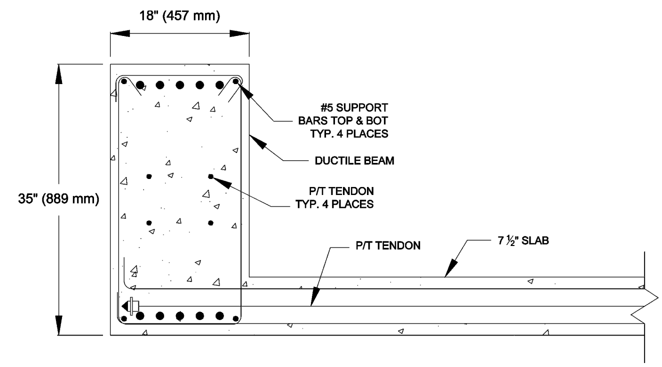

Fig. 1—Building 1 typical ductile beam detail as designed.

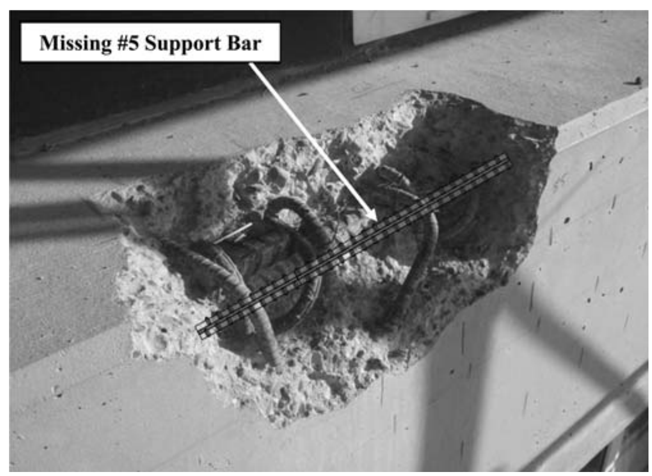

Fig. 2—Building 1 ductile beam as built.

To study the effect of improperly anchored stirrups on the shear strength of reinforced concrete beams, two 16 ft (4.9 m) long beams with cross sections of 13 x 24 in. (330 x 610 mm) and a test region on each end were fabricated for a total of four tests, each with a shear span-depth ratio (a/d) of approximately 3. Each of the four test regions had a different stirrup configuration, with one control test and three variations of improperly anchored shear reinforcement. These reinforcement details were similar to those found in Buildings 1 and 2 during past WDP investigations. ACI 318-08[1]

ACI Structural Journal, V. 108, No. 4, July-August 2011.

MS No. S-2010-082.R1 received August 31, 2010, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published in the May-June 2012 *ACI Structural Journal* if the discussion is received by January 1, 2012.

ACI member **Jason C. Varney** *is a Structural Engineer-in-Training (EIT) at Huitt-Zollars, Inc., Dallas, TX. He received his BSCE in 2008 from Tufts University, Medford, MA, and his MSE from the University of Texas at Austin, Austin, TX, in 2009.*

ACI member **Michael D. Brown** *is a Staff Engineer at WDP & Associates, PC, Austin, TX. He received his BSCE, MSE, and PhD in 2000, 2002, and 2005, respectively, from the University of Texas at Austin. He is a member of ACI Committees 228, Nondestructive Testing of Concrete, and Joint ACI-ASCE Committees 423, Prestressed Concrete, and 445, Shear and Torsion.*

Oguzhan Bayrak, *FACI, is an Associate Professor in the Department of Civil, Environmental, and Architectural Engineering and holds the Charles Elmer Rowe Fellowship in Engineering at the University of Texas at Austin, where he serves as Director of the Phil M. Ferguson Structural Engineering Laboratory. He is a member of ACI Committees 341, Earthquake-Resistant Concrete Bridges, and S803, Faculty Network Coordinating Committee, and Joint ACI-ASCE Committees 441, Reinforced Concrete Columns, and 445, Shear and Torsion.*

Randall W. Poston, *FACI, is a Principal at WDP & Associates, PC, and received his engineering degrees from the University of Texas at Austin. He is Chair of ACI Committee 318, Structural Concrete Building Code. He is a past member of the ACI Board of Direction and TAC and past Chair of the Technical Activities Committee (TAC) Repair and Rehabilitation Committee and ACI Committee 224, Cracking. He is a member of ACI Committees 222, Corrosion of Metals in Concrete, and 562, Evaluation, Repair, and Rehabilitation of Concrete Buildings.*

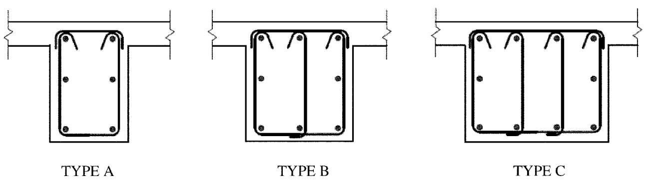

Fig. 3—Typical Building 2 stirrup details as shown in contract drawings.

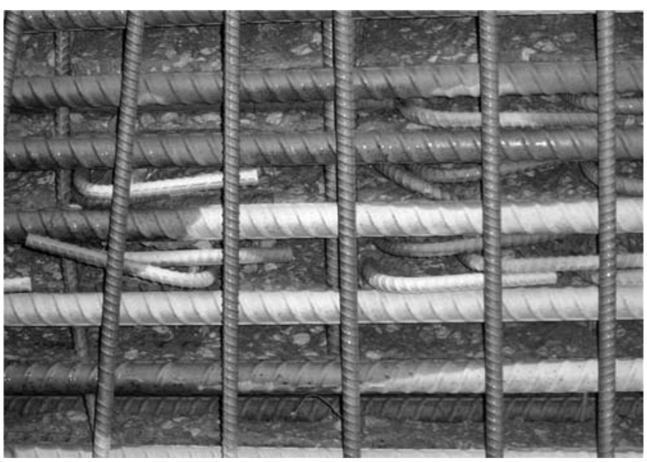

Fig. 4—Unanchored center stirrup legs in Building 2 (view of bottom of beam).

and AASHTO LRFD Bridge Design Specification[2,3] provisions were used to compare the resulting failure loads from tests with calculated nominal capacities, and conclusions were drawn on the relationship between these particular improper stirrup details and the shear strength of reinforced concrete beams. It should be noted that although the work outlined in this paper is motivated by questionable stirrup details found in Buildings 1 and 2, the intention is not to accurately model these conditions but rather to examine similar reinforcement arrangements.

RESEARCH SIGNIFICANCE

Current ACI 318-08[1] and ACI 315-99[4] provisions regarding the anchorage of shear reinforcement are vague and, similar to all code provisions, have evolved over time.

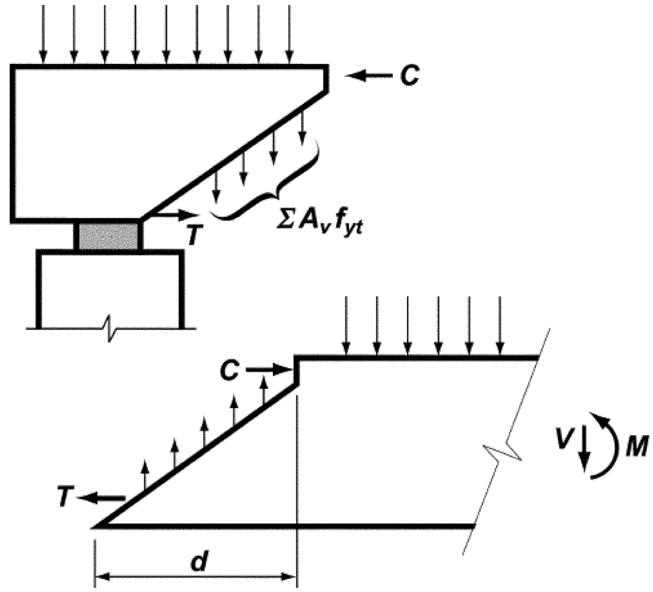

Fig. 5—Free-body diagrams of end of beam (ACI 318-08,[1] Fig. R11.1.3.1a).

Although it is not uncommon to encounter improperly constructed structures in practice, there is little information in the literature regarding the performance of beams with improperly anchored shear reinforcement. The objective of this study was to: 1) examine the impact of improperly anchored shear reinforcement; and 2) determine the reduction in capacity associated with the questionable reinforcement details tested.

BACKGROUND: SHEAR STRENGTH

The traditional approach to shear strength of reinforced concrete members incorporates the assumption that the nominal shear capacity of a reinforced concrete section V_n is composed of two components: the resistance provided by the concrete V_c and the resistance provided by the shear reinforcement V_s (Fig. 5).

Shear strength, as calculated in ACI 318-08,[1] is based on an average shear stress on the full effective cross section, and the code equations assume that the shear strength provided by concrete V_c is the same for beams with and without shear reinforcement. If no reinforcement is present in a concrete member, the shear strength is a function of the diagonal tensile strength of the concrete. Once this strength is exceeded, a shear crack forms and the beam fails rapidly.

If shear reinforcement is provided, the concrete contribution to shear strength in a reinforced concrete member is a function of both the aggregate interlock and the dowel action between the concrete and reinforcement. Once a shear crack forms, the shear reinforcement engages, providing the remaining shear capacity of the section V_s. For the shear reinforcement to develop its full capacity, stirrups must be properly anchored.

CODE PROVISIONS: A HISTORICAL PERSPECTIVE

It is commonly accepted practice to detail a beam cross section such that a longitudinal bar is placed in each corner of a closed stirrup and at the hooked ends of single-leg stirrups. Cross sections are detailed as such for several reasons, including:

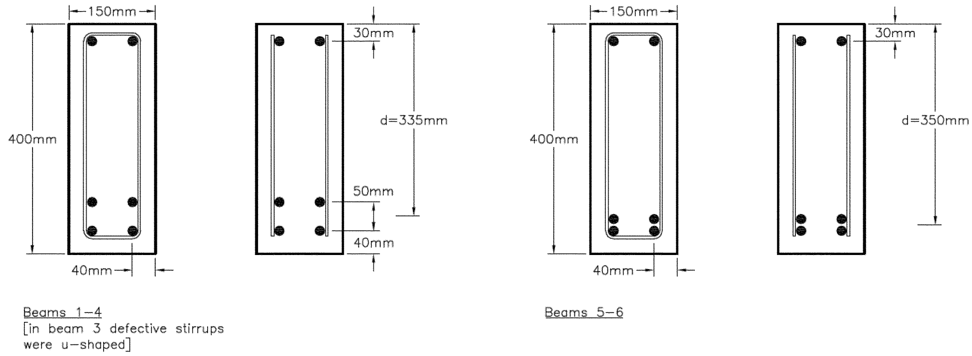

Fig. 6—Typical details tested by Regan and Kennedy Reid.[10] Control specimens (closed stirrups) and simulated loss of anchorage due to corrosion (straight bars).

- Constructability purposes;
- Prevention of presumed concrete crushing at the corner of the stirrup, resulting from the high stress concentrations that develop in this region when the member is loaded; and
- To secure a positive anchorage to develop the full capacity of the shear reinforcement.

Despite the seemingly intuitive assumptions supporting the latter two arguments, historical code requirements do not appear to substantiate these notions.

As far back as 1910, the anchorage of shear reinforcement has been explicitly discussed in code language. The 1910 National Association of Cement Users' "Standard Building Regulations for the Use of Reinforced Concrete," Section 73,[5] requires that "members of web reinforcement shall be embedded in the compression portion of the beam so that adequate bond strength is provided to fully develop the assumed strength of all shear reinforcement." With respect to stirrup anchorage on the tension side, the document states that "web reinforcement, unless rigidly attached, shall be placed at right angles to the axis of the beam and looped around the extreme tension member."

The 1920 follow-up to this document, ACI Standard Specifications No. 23, Section 44(f),[6] echoes this notion, stating that "in case the end anchorage is not in bearing on other reinforcing steel, the anchorage shall be such as to engage an adequate amount of concrete to prevent the bar from pulling off a portion of the concrete." Although these documents require hooking stirrups around flexural reinforcement on the tension side, the objective is to ensure that the full capacity of the bar is developed rather than to prevent concrete crushing within the hook.

ACI 318-41[7] presented the first explicit anchorage requirements for shear reinforcement. Single-leg stirrups were required to:

- Be welded to the longitudinal reinforcement;
- Be tightly hooked (180 degrees) around the longitudinal reinforcement;
- Be embedded such that a maximum bond stress is not attained; or
- Have a standard hook, considered to develop 10,000 psi (69 MPa), plus sufficient embedment to develop the remainder of the stress to which it is subjected by bond stress.

ACI 318-41[7] was also the first document to explicitly require that both the extreme and closed ends of simple U-stirrups must be hooked around the longitudinal reinforcement to be considered effectively anchored. Again, the shear reinforcement anchorage is mentioned in the context of developing the full capacity of the shear reinforcement by bond stress. These requirements remained the same until ACI 318-63,[8] which allowed the development of more stress to be considered by a standard hook alone and provided specific embedment requirements for the full capacity of a stirrup to be developed.

It was not until ACI 318-89[9] that single-leg stirrups (No. 5 bar or smaller; No. 6, No. 7, or No. 8 bars if $f_y \leq 40{,}000$ psi [275.79 MPa]) were required to be anchored around a longitudinal bar with a standard hook. Although wrapping stirrups around the longitudinal reinforcement is accepted as sufficient anchorage for smaller bars, ACI 318-08[1] recognizes that it is not possible to bend larger bars tightly around longitudinal steel and embedment requirements still govern. The option of a straight anchorage was removed, as it is difficult to hold the bar in place while casting and the lack of a hook may render the stirrup ineffective as it crosses shear cracks near the end of the stirrup. As seen through historical code development, however, the primary anchoring mechanism for shear reinforcement is obtained through the bond developed along the length of the bar. Although it provides advantages, the requirement to anchor the stirrup around a longitudinal bar is more of a code simplification than a structural necessity. Given adequate embedment length, a straight bar could perform equally as well as a similarly sized stirrup hooked around the longitudinal reinforcement.

PREVIOUS RESEARCH

There is little information in the literature dealing directly with the effect of improperly anchored shear stirrups, especially as detailed in this study. Relevant work, however, was performed by Regan and Kennedy Reid[10] regarding the effects of corrosion in the end anchorages of undeformed shear reinforcement. Regan and Kennedy Reid[10] modeled the deterioration of shear reinforcement by replacing normal closed stirrups with straight bars ("defective stirrups") at varying intervals along the length of several test specimens (Fig. 6). Each test specimen was simply supported and subjected to a concentrated load at midspan.

The researchers[10] found that shear capacities were reduced by 14 to 33% for a 65 to 75% loss in anchorage in specimens designed to fail in shear, whereas test specimens designed to yield in flexure (with compromised stirrup anchorages) developed their full flexural capacity M_n. It should be noted that removal of anchorage in the context of this study was removal of the standard hook itself, not solely anchorage

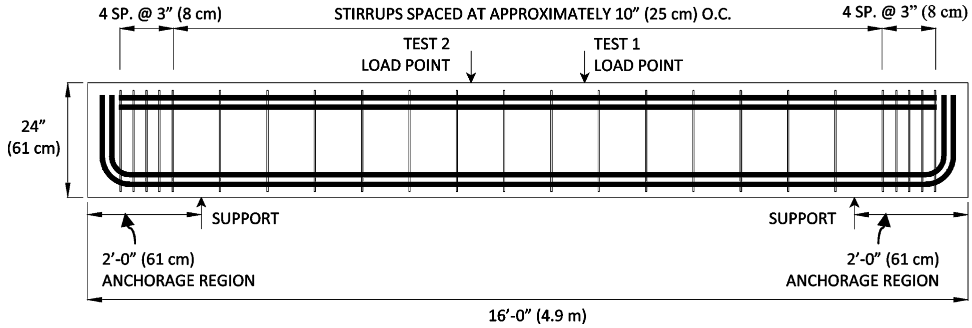

Fig. 7—Test specimen elevation.

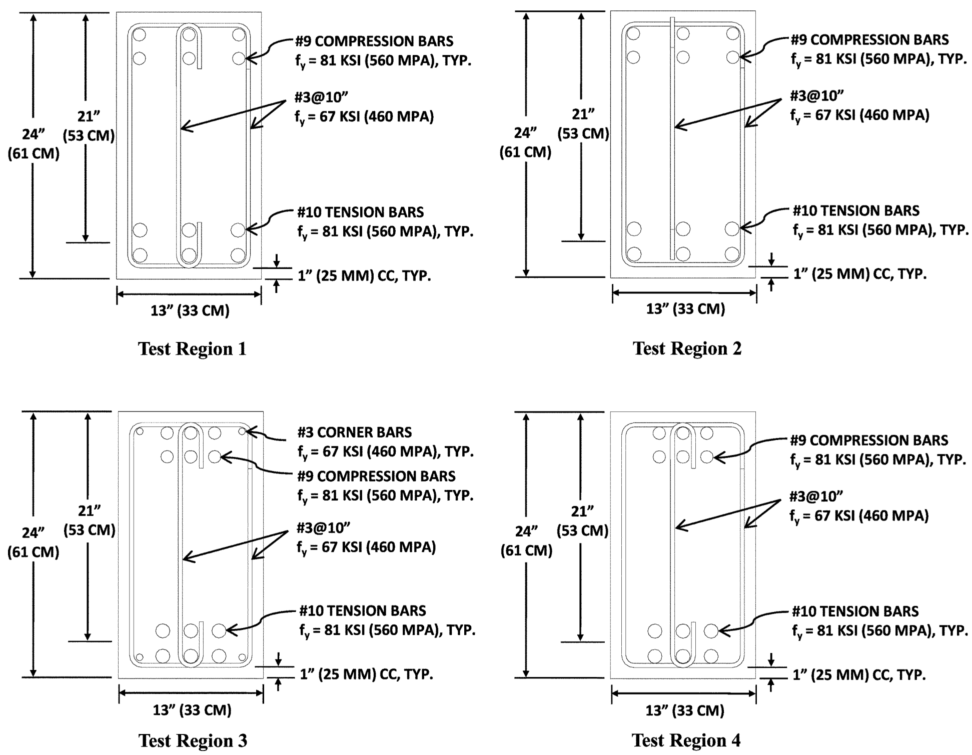

Fig. 8—Test specimen cross sections.

around the longitudinal steel. It is evident that stirrups lacking end anchorages make a significant contribution to the shear capacity of reinforced concrete sections, even while relying solely on the bond stress developed along the straight length of the bar. One can presume that the presence of an intact standard hook on these stirrups, whether anchored around a longitudinal bar or not, would have further increased the shear capacity of the sections.

EXPERIMENTAL PROGRAM

Specimen preparation

The specimen design was limited by the need to conform to the dimensions and load capacity of an existing reaction frame in the Ferguson Structural Engineering Laboratory (FSEL). An a/d of approximately 3 was chosen, enabling two tests per beam while still ensuring a beam shear failure. Based on the results of a database of previous concrete shear tests compiled by Brown et al.,[11] over 98% of specimens with an a/d of 3 experienced shear failure below $10\sqrt{f_c'}$. Therefore, the specimens were designed such that the flexural capacity would result in a calculated shear stress between $9\sqrt{f_c'}$ and $10\sqrt{f_c'}$, a value significantly higher than the calculated nominal capacity ($V_c + V_s$). Two 16 ft (4.9 m) long, 13 x 24 in. (330 x 610 mm) beams were designed to ACI 318-08[1] strength and detailing provisions. Each specimen included a 2 ft (24 in.) anchorage zone at both ends for the longitudinal reinforcement, resulting in a 12 ft (3.7 m) simple span beam (Fig. 7).

Each beam test region had a different shear reinforcement detail, all of which are summarized in Fig. 8. Test Region 1, the control condition, had properly anchored center stirrup legs and flexural reinforcement in the corners of the closed stirrups. Test Region 2 was detailed similar to the beams found in Building 2, with the center stirrup leg not properly anchored around a longitudinal bar on the tension side. Test Region 3 was detailed to evaluate the effect of corner bar size on the capacity of the beams, as the flexural steel was moved toward the middle of the cross section and No. 3 bars were used in the corners, similar to the beam reinforcement detail in Building 1. Test Region 4 was detailed similar to the as-built beams in Building 1, with flexural steel concentrated in the middle of the cross section and no longitudinal bars in the corners of the closed stirrups. All stirrups and flexural reinforcement consisted of uncoated

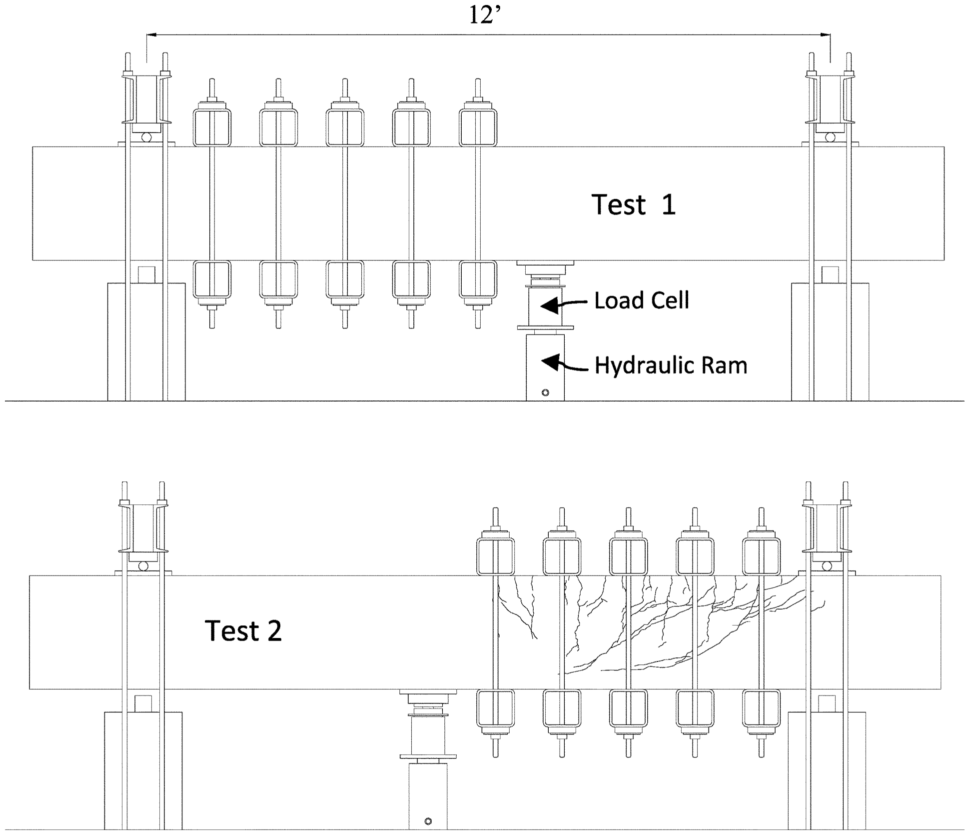

Fig. 9—Test frame: schematic.

deformed bars, and a stirrup height-to-stirrup diameter ratio of 58.7 was typical in each test region.

Specimen loading

Specimens were tested upside down, with a setup providing a simple span of 12 ft (3.7 m). In each of the four tests, the specimen was loaded in approximately 15 kip (67 kN) increments with a single hydraulic ram. Upon completion of a loading stage, the load was held fixed and observations were noted regarding crack locations, extensions, and widths. Because two tests were conducted on each specimen, the half of the beam not being tested was precompressed with five tensioned-rod clamps that essentially acted as external stirrups. Note that minor cracking in the shear span is believed to have a negligible, if any, effect on the shear capacity of the section.[12] The test setup and clamps are shown in Fig. 9 and 10.

Instrumentation

Strain gauges were used to monitor the elongation of both the shear and flexural reinforcement during loading. Strain gauges were attached to the shear reinforcement at the center of the stirrups near the middle of the shear span (Fig. 11). Strain in the flexural reinforcement was monitored at the load point on the tension side of the beam (Fig. 12).

Concrete placement

Both beams were cast using the same batch of commercially available, ready mixed concrete with a 3/8 in. (10 mm) maximum aggregate size. The concrete mixture design is summarized in Table 1. Specimens were cast in three separate lifts. The measured slump was 4.5 in. (110 mm) (ASTM C143/C143M-08[13]) and 4 x 8 in. (100 x 200 mm) cylinders were cast to measure the concrete strength during testing.

Fig. 10—Test frame: photograph.

Material testing

The cylinders were tested to determine the concrete compressive strength at 7, 14, 21, and 28 days from casting, as well as on each test day. The cylinders were capped with unbonded neoprene pads conforming to ASTM C1231/C1231M-09[14] and the concrete compressive strength was measured per ASTM C39/C39M-05[15] specifications. To obtain each desired compressive strength, three 4 x 8 in. (100 x 200 mm) cylinders were tested. The resulting compressive strengths were averaged to obtain the final compressive strength unless one of the three was noticeably inconsistent with the others, in which case the inconsistent result was neglected and the remaining two were averaged.

The yield strength of the steel reinforcement was determined according to ASTM A370-09[16] using 4 ft (1.2 m) long tension coupons. Three tests were performed for each different bar size.

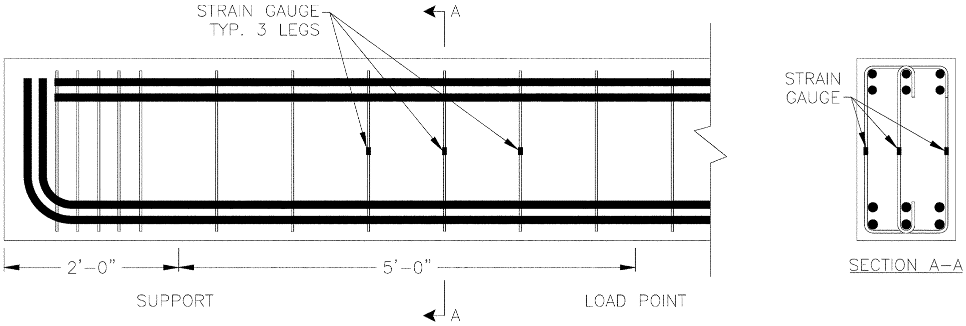

Fig. 11—Elevation and section views of shear reinforcement instrumentation (instrumentation symmetric about center of beam, typical for both beams).

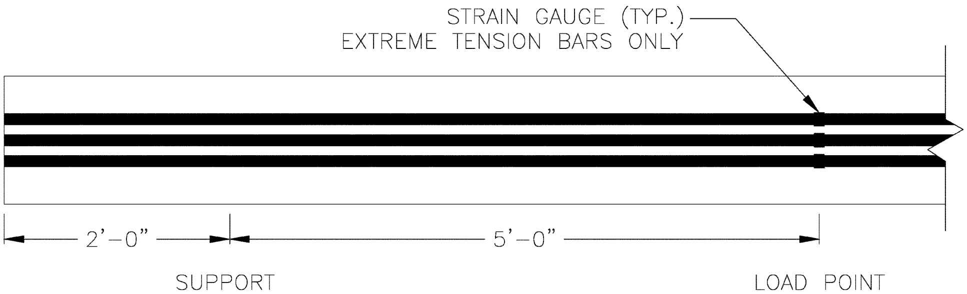

Fig. 12—Plan view of flexural reinforcement instrumentation (instrumentation symmetric about center of beam, typical for both beams).

Table 1—Concrete mixture design

Fine aggregate (sand), lb/yd^3 (kg/m^3)	1657 (983)
Coarse aggregate (pea gravel), lb/yd^3 (kg/m^3)	1726 (1024)
Cement, lb/yd^3 (kg/m^3)	320 (190)
Fly ash, lb/yd^3 (kg/m^3)	83 (49)
Water, lb/yd^3 (kg/m^3)	125 (74)
w/c	0.39
Retarder, lb/yd^3 (kg/m^3)	0.4 (0.2)
Water reducer, lb/yd^3 (kg/m^3)	1.0 (0.6)

The Grade 60 bars yielded at 67 ksi (462 MPa) and the Grade 75 bars yielded at 81 ksi (558 MPa). The results of the material strength tests are summarized in Table 2.

EXPERIMENTAL OBSERVATIONS

The key parameters and results from each test are summarized in Table 2. In general, each of the four tests produced comparable results. Visual observations, verified with data obtained from the test instrumentation, show that a shear failure was obtained in each of the four tests. It was assumed that failure was achieved when the specimen stopped gaining load and a load drop of at least 10 kips (45 kN) was observed. A free-body diagram is shown in Fig. 13, where R_A and R_B are the support reactions, P_{RB} is the force due to the self-weight of the reaction beams, and $P_{APPLIED}$ is the load applied by the hydraulic ram.

The post-failure crack patterns (Fig. 14 through 17) were similar in all four tests, and each displays a series of cracks characteristic of a shear failure. Each test exhibited a typical progression from flexure cracks to flexure-shear cracking.

Load-deflection data at the load point further support the observation that shear failure was obtained in each test. Data for each test exhibit a noticeable and sudden drop in load, characteristic of a typical nonductile shear failure. Load-deflection plots for the load point for each test are shown in Fig. 18.

Flexural yielding was not encountered in any of the four tests based on the measured longitudinal bar strains. Linear-elastic behavior in the longitudinal reinforcement can be verified by observing the linearity of the typical load versus strain plots for the longitudinal reinforcement shown in Fig. 19.

The first shear cracks were observed at a noticeably lower load in Specimen 1 (Tests 1 and 2) than in Specimen 2 (Tests 3 and 4). Diagonal cracks formed at loads of 50 and 51 kips (222 and 227 kN) in the first two tests, as compared to the second two tests, where diagonal cracks were observed at 74 and 101 kips (329 and 449 kN). These cracks first appeared as small hairline cracks, opening and extending gradually in all cases except Test 4, where the specimen experienced a 3 kip (13 kN) load drop and a crack opened suddenly across nearly the entire shear span.

CALCULATION OF SHEAR STRENGTH

Nominal capacities for the control section were calculated using four well-known shear capacity methods: the ACI 318-08[1] simplified and detailed methods, as well as the general modified compression field approaches outlined in the AASHTO LRFD 2007 Bridge Design Specifications[2] and AASHTO LRFD 2007 Bridge Design Specifications with 2008 Interim Revisions.[3] The shear load at failure of each of the four specimens exceeded the greatest calculated nominal capacity. A comparison of the nominal shear capacities (using the average material strengths for all four tests) with failure loads is presented in Fig. 20, whereas the capacities and failure loads for each individual test are summarized in Table 3. The shear capacities are calculated at a distance *d* from the face of the support.

Table 2—Summary of test results

Test	Anchorage detail	f_c', psi (kPa)	$f_{y,Gr60}$, psi (kPa)	$f_{y,Gr75}$, psi (kPa)	Applied shear at failure, kip (kN)
Test Region 1	Control	3610 (24,900)	67,000 (462,000)	81,000 (558,000)	130 (578)
Test Region 2	Unanchored center leg	3780 (26,100)	67,000 (462,000)	81,000 (558,000)	125 (556)
Test Region 3	No. 3 corner bars	3640 (25,100)	67,000 (462,000)	81,000 (558,000)	128 (569)
Test Region 4	No corner bars	3910 (27,000)	67,000 (462,000)	81,000 (558,000)	149 (663)

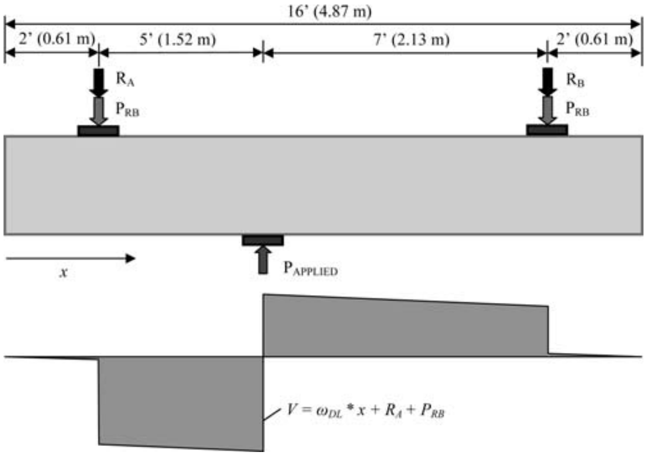

Fig. 13—Free-body diagram.

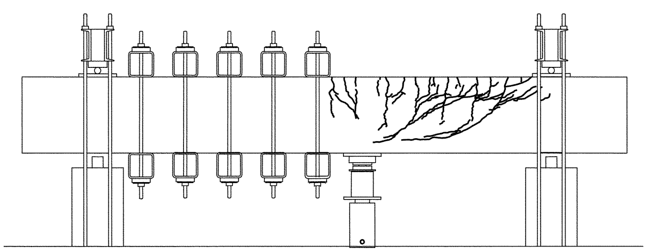

Fig. 14—Test 1 post-failure crack pattern.

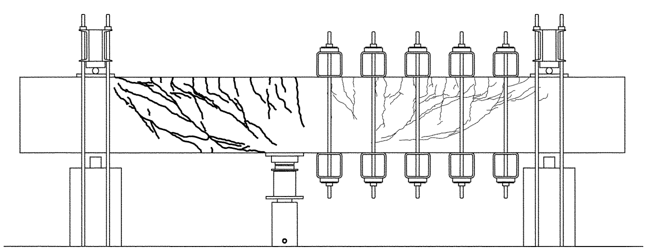

Fig. 15—Test 2 post-failure crack pattern (pre-existing cracks on right).

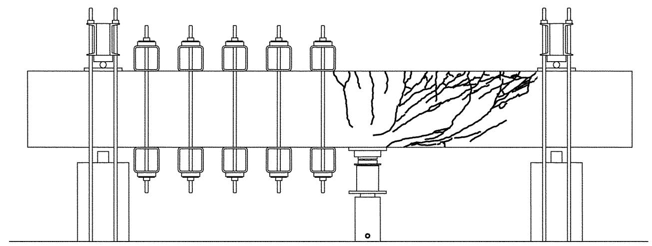

Fig. 16—Test 3 post-failure crack pattern.

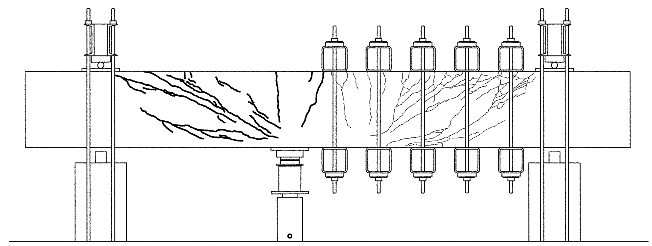

Fig. 17—Test 4 post-failure crack pattern (pre-existing cracks on right).

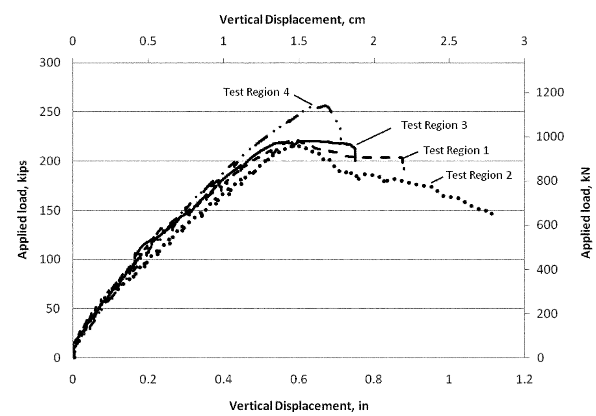

Fig. 18—Average load-deflection responses for each specimen.

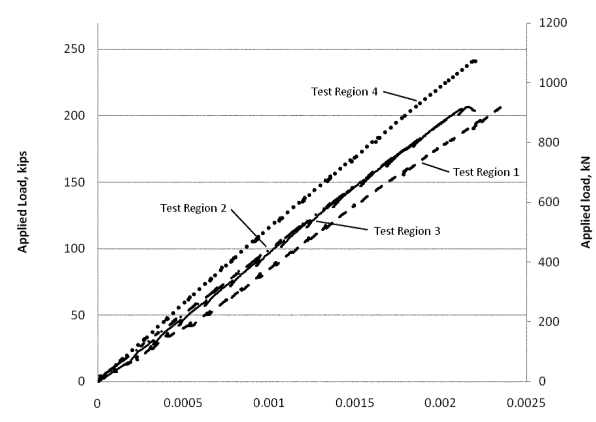

Fig. 19—Average load versus strain in flexural reinforcement for each specimen.

DISCUSSION OF RESULTS

The resulting failure loads, compared to the capacities calculated using various code methods, are summarized in Fig. 20. The failure loads for Tests 1, 2, and 3 exhibited a high degree of consistency, as the shear loads at failure fell within 4% of each other at 130, 125, and 128 kips (578, 556, and 569 kN), respectively. Contrary to the low scatter among failure loads in the first three tests, failure occurred at a noticeably higher shear load of 149 kips (663 kN) in Test 4. Although it is within the anticipated range of scatter for shear testing, the failure load for Test 4 falls outside a single standard deviation for the data set.

Effect of presence and size of corner bars

The presence of longitudinal bars at the corners of the closed stirrups appeared to have no discernable effect on the shear capacity of the sections. Three different corner bar configurations were tested (Tests 1, 2, 3, and 4), and there was no evidence in the data that the lack of longitudinal bars in the stirrup corners led to any decrease in shear capacity. In fact, the only noticeable difference in shear capacity was observed in the beam end with no corner bars, with a failure load over 30 kips (133 kN) higher than the control test region.

Similarly, the size of the corner bars, when present, had no conclusive effect on the shear capacity of the sections. Comparing Tests 1 and 3, essentially the same shear capacity was obtained in the beam end using a No. 3 corner bar as the beam end with No. 9 and No. 10 corner bars.

Effect of stirrup anchorage

There was no observed difference in shear capacity based on whether or not the tension-side end of the single-leg stirrup was properly anchored around a longitudinal bar. The strain gauge data for Test 2 show that the unanchored center leg actually strained more at lower loads than the outside stirrups in the portion of the test region near the support (Fig. 21).

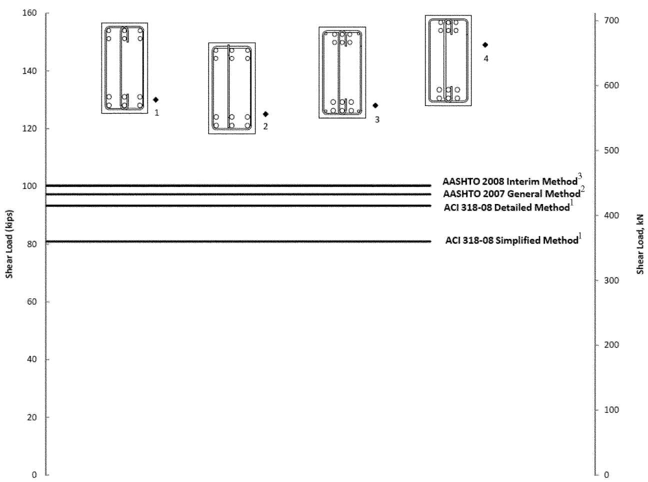

Fig. 20—Comparison of nominal capacities to failure loads.

RECOMMENDATIONS

The results of this study suggest that the anchorage of shear reinforcement, as tested, does not have a significant effect on the shear capacity of a reinforced concrete section. The design or construction of questionable reinforcement details, however, is certainly not recommended. Previous assumptions that have led to current ACI 318-08[1] design provisions concerning the anchorage of shear reinforcement are practical and make intuitive sense, and because their implementation does not have an adverse effect on shear capacity, these requirements should be observed. If questionable anchorage deficiencies are discovered in the field and a repair design is considered, however, the results of this study suggest that a significant amount of shear capacity is still retained and large-scale repairs may not be necessary.

Further research could be performed to evaluate the effect of varying beam dimensions, stirrup sizes, and concrete strengths. Additionally, it would be beneficial to evaluate these effects under reversed cyclic loading to better understand the performance of similar sections under seismic conditions. The presence of concrete crushing at the stirrup corners may be more pronounced in such tests, and it would be advantageous to engineers to further understand the mechanics of load transfer in such cases.

CONCLUSIONS

Based on the performance of the four specimens, the anchorage of shear reinforcement by longitudinal reinforcing bars, as tested, does not affect the shear capacity of a reinforced concrete section. Stirrups, as detailed in this

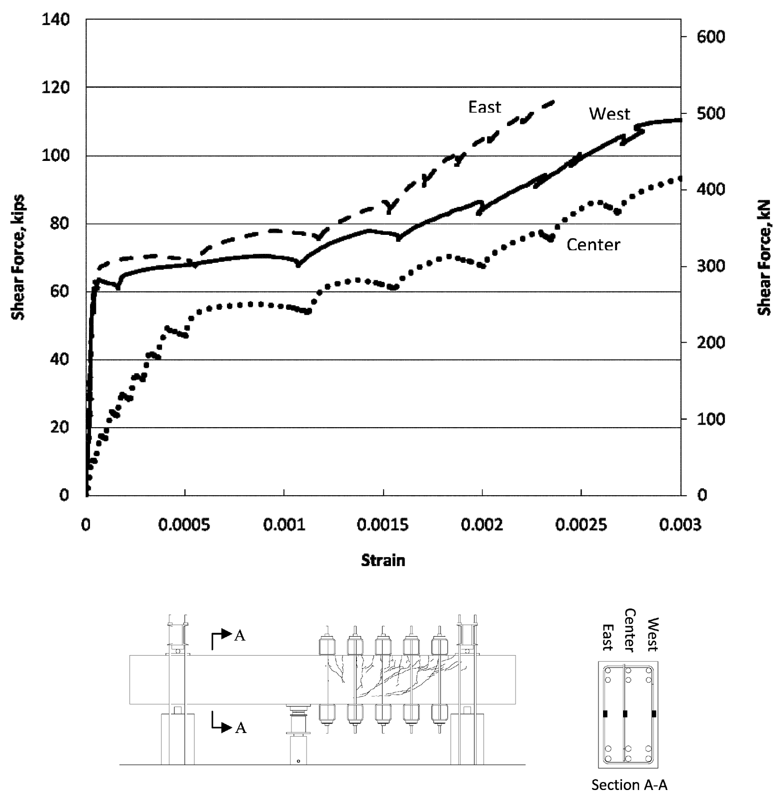

Fig. 21—Test Region 2 load versus strain for stirrups near support.

Table 3—Summary of nominal shear capacities

Test	f_c', psi (kPa)	ACI simplified,[1] kip (kN)	ACI detailed,[1] kip (kN)	AASHTO 2007,[2] kip (kN)	AASHTO 2008 Int.,[3] kip (kN)
Test Region 1	3610 (24,900)	79 (351)	97 (431)	93 (414)	100 (445)
Test Region 2	3780 (26,100)	80 (356)	97 (431)	93 (414)	102 (454)
Test Region 3	3640 (25,100)	79 (351)	97 (431)	93 (414)	101 (449)
Test Region 4	3910 (27,000)	81 (360)	98 (436)	94 (418)	98 (436)

study, appear to engage due to bond stresses developed along the length of the bar and standard hook rather than due to anchorage around the flexural reinforcement. Despite this fact, it remains good practice for both contractors and designers to observe the reinforcement anchorage requirements present in ACI 318-08,[1] AASHTO LRFD,[2,3] and other design codes. In other words, the conclusions of this study are meant to shed light on the structural evaluation of existing building components when shear reinforcement anchorage has perceived deficiencies—they are not to be taken as recommendations for use in new construction. Additionally, it is not the intention of the authors to extrapolate the results to either Buildings 1 or 2 without additional engineering judgment and analysis. It is hoped that the tests reported herein demonstrate that in the event as-constructed beams in service are discovered without customary longitudinal bar anchorage details, responsible professionals will consider the need, if any, of requiring retrofit strengthening and the financial implications of effecting such strengthening.

ACKNOWLEDGMENTS

Financial support for this project was provided by the Ferguson Structural Engineering Laboratory (FSEL) at the University of Texas at Austin, Austin, TX, and Whitlock Dalrymple Poston & Associates, PC, Austin, TX.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 2008, 473 pp.

2. AASHTO, "LRFD Bridge Design Specifications," fourth edition, American Association of State Highway and Transportation Officials, Washington, DC, 2007, 1520 pp.

3. AASHTO, "LRFD Bridge Design Specifications," (fourth edition with 2008 interim revisions), American Association of State Highway and Transportation Officials, Washington, DC, 2008, 1520 pp.

4. Joint ACI-CRSI Committee 315, "Details and Detailing of Concrete Reinforcement (ACI 315-99)," American Concrete Institute, Farmington Hills, MI, 1999, 44 pp.

5. NACU Standard No. 4, "Standard Building Regulations for the Use of Reinforced Concrete," *Proceedings, National Association of Cement Users*, V. 6, Feb. 1910, Philadelphia, PA, 1910, pp. 349-361.

6. ACI Standard Specifications No. 23, "Standard Building Regulations for the Use of Reinforced Concrete," ACI JOURNAL, *Proceedings* V. 16, No. 2, Feb. 1920, pp. 283-302.

7. ACI Committee 318, "Building Regulations for Reinforced Concrete (ACI 318-41),"American Concrete Institute, Farmington Hills, MI, 1941, 67 pp.

8. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318R-63)," American Concrete Institute, Farmington Hills, MI, 1963, 144 pp.

9. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318R-89) and Commentary," American Concrete Institute, Farmington Hills, MI, 1989, 353 pp.

10. Regan, P. E., and Kennedy Reid, I. L., "Shear Strength of RC Beams with Defective Stirrup Anchorages," *Magazine of Concrete Research*, V. 56, No. 3, 2004, pp. 159-166.

11. Brown, M. D.; Bayrak, O.; and Jirsa, J. O., "Design for Shear Based on Loading Conditions," *ACI Structural Journal*, V. 103, No. 4, July-Aug. 2006, pp. 541-550.

12. Birrcher, D. et al., *Strength and Serviceability Design of Reinforced Concrete Deep Beams*, Research Report 5253-1, Center for Transportation Research, University of Texas at Austin, Austin, Texas, 2009, 400 pp.

13. ASTM C143/C143M-08, "Standard Test Method for Slump of Hydraulic-Cement Concrete," ASTM International, West Conshohocken, PA, 2008, 4 pp.

14. ASTM C1231/C1231M-09, "Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders," ASTM International, West Conshohocken, PA, 2009, 5 pp.

15. ASTM C39/C39M-05, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," ASTM International, West Conshohocken, PA, 2005, 7 pp.

16. ASTM A370-09, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products," ASTM International, West Conshohocken, PA, 2009, 47 pp.

APPENDIX A—SHEAR CAPACITY SAMPLE CALCULATIONS

Shear capacity calculations for ACI 318-08[1] simplified method

$$V_n = V_c + V_s \tag{11-2}$$

$$V_c = 2\sqrt{f_c'}\,bd \tag{11-3}$$

$$V_c = 2\sqrt{3610\text{ psi}}\,(13\text{ in.})(21\text{ in.}) = 32.8\text{ kip}$$

$$V_s = \frac{A_v f_{yt} d}{s} \tag{11-15}$$

$$V_s = \frac{(0.33\text{ in.}^2)(67\text{ ksi})(21\text{ in.})}{(10\text{ in.})} = 46.4\text{ kip}$$

$$V_n = 32.8k + 46.4k = 79.2\text{ kip}$$

Shear capacity calculations for ACI 318-08[1] detailed method

$$V_n = V_c + V_s \tag{11-2}$$

$$V_c = \left(1.9\lambda\sqrt{f_c'} + 2500\rho_w\frac{V_u d}{M_u}\right)b_w d,\quad \frac{V_u d}{M_u} \le 1 \tag{11-5}$$

$$V_c = \Big[(1.9)(1)\sqrt{3610\text{ psi}} + 2500\left(\frac{7.62\text{ in.}^2}{(13\text{ in.})(21\text{ in.})}\right)(1)\Big](13\text{ in.})(21\text{ in.}) = 50.2\text{ kip}$$

$$V_s = \frac{A_v f_{yt} d}{s} \tag{11-15}$$

$$V_s = \frac{(0.33\text{ in.}^2)(67\text{ ksi})(21\text{ in.})}{(10\text{ in.})} = 46.4\text{ kip}$$

$$V_n = 50.2k + 46.4k = 96.6\text{ kip}$$

Shear capacity calculations for AASHTO simplified method (AASHTO LRFD Bridge Design Specifications, 2008 Interim Revisions[3])

$$V_n = V_c + V_s + V_p \quad (5.8.3.3\text{-}1)$$

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \quad (5.8.3.3\text{-}3)$$

$$\beta = 4.8/(1 + 750\varepsilon_s) \quad (5.8.3.4.2\text{-}1)$$

$$\varepsilon_s = \left(\frac{|M_u|/d_v + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}}{E_sA_s + E_pA_{ps}}\right) \quad (5.8.3.4.2\text{-}4)$$

$$\varepsilon_s = \left(\frac{(227\text{ k/ft})(12\text{ in./ft})/18.9\text{ in.} + 0 + |130k - 0| - 0}{(29{,}000\text{ ksi})(7.62\text{ in.}^2) + 0}\right)$$

$= 0.00124$ in./in.

$\beta = 4.8/(1 + 750(0.00124\text{ in./in.})) = 2.48$

$V_c = 0.0316(2.48)\sqrt{3.61\text{ ksi}}\,(13\text{ in.})\,(18.9\text{ in.}) = 36.7\text{ kip}$

$$V_s = (A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha)/s \quad (5.8.3.3\text{-}4)$$

$$\theta = 29 + 3500\varepsilon_s \quad (5.8.3.4.2\text{-}3)$$

$$\theta = 29 + 3500(0.00124) = 33.3°$$

$$V_s = [(0.33\text{ in.}^2)(67\text{ ksi})(18.9\text{ in.}) \times (\cot 33.3° + \cot 90°)\sin 90°]/10\text{ in.} = 63.5\text{ kip}$$

$$V_n = 36.7k + 63.5k + 0 = 100.2\text{ kip}$$

Shear capacity calculations for AASHTO general method (AASHTO LRFD Bridge Design Specifications[2])

$$V_n = V_c + V_s + V_p \quad (5.8.3.3\text{-}1)$$

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \quad (5.8.3.3\text{-}3)$$

β and θ determined iteratively using Table 5.8.3.4.2-1

$$v_u/f'_c = \frac{130k}{(13\text{ in.})(18.9\text{ in.})}/3.61\text{ ksi} = 0.147$$

$$\varepsilon_x = \left(\frac{|M_u|/d_v + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}\right) \quad (5.8.3.4.2\text{-}1)$$

$$\varepsilon_x = \left(\frac{(227\text{ k/ft})(12\text{ in./ft})/18.9\text{ in.} + 0 + 0.5|130k - 0|\cot 34.9° - 0}{2[(29{,}000\text{ ksi})(7.62\text{ in.}^2)] + 0}\right)$$

$= 0.00054$ in./in.

$\beta = 2.21$

$\theta = 34.9°$

$V_c = 0.0316(2.21)\sqrt{3.61\text{ ksi}}(13\text{ in.})(18.9\text{ in.}) = 32.6\text{ kip}$

$$V_s = (A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha)/s \quad (5.8.3.3\text{-}4)$$

$$V_s = [(0.33\text{ in.}^2)(67\text{ ksi})(18.9\text{ in.}) \times (\cot 34.9° + \cot 90°)\sin 90°]/10\text{ in.} = 59.9\text{ kip}$$

$$V_n = 32.6k + 59.9k = 92.5\text{ kip}$$